

REVIEW OF STRUCTURAL STABILITY

NORTH SPRINGFIELD DAM

CONNECTICUT RIVER BASIN

BLACK RIVER, VERMONT

NOVEMBER 1977

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASS.

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NORTH SPRINGFIELD LAKE

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SUMMARY OF REPORT

A stability analysis of the principal concrete structures of North Springfield Dam was performed to determine whether these structures satisfy current design criteria. The structural elements considered are as listed:

- (a) Spillway Weir
- (b) Right Approach Channel Wall
- (c) Left Approach Channel Wall
- (d) Intake Structure
- (e) Service Bridge Piers and Abutment

The analysis indicates that all of the structures have adequate stability in accordance with the prescribed criteria and do not require any modifications.

REVIEW OF STRUCTURAL STABILITY

NORTH SPRINGFIELD DAM

PART I

GENERAL DESCRIPTION

1.1 Purpose

The objective of this study is to review the stability of the principal concrete structures, based upon current criteria in cases where the original design criteria were less conservative. This review has been performed to comply with Corps of Engineers regulation ER-1110-2-100, Periodic Inspection and Continuing Evaluation of Completed Civil Works Structures (30 March 1977).

1.2 Stability Criteria

The current stability criteria by which this project was evaluated are contained in the following Corps of Engineers publications:

Engineering Manuals:

EM 1110-2-2101, Working Stresses for Structural Design (17 Jan 1972)
EM 1110-2-2220, Gravity Dam Design (23 Nov 1960)
EM 1110-2-2400, Structural Design of Spillways and Outlet Works
(2 Nov 1964)
EM 1110-2-2501, Wall Design: Flood Walls (18 June 1962)
EM 1110-2-2502, Retaining Walls (25 Jan 1965)

Engineer Technical Letters:

ETL 1110-2-184, Gravity Dam Design (25 Feb 1974)

Engineering Regulations:

ER 1110-2-1806, Earthquake Design and Analysis for Corps of Engineers Dams (30 Apr 1977)

1.3 Pertinent References

Pertinent data, computations and drawings are contained in the following:

Project Data for Periodic Inspection - North Springfield Lake (Aug 1972)
Design Memorandum No. 2 - General - North Springfield Dam & Reservoir
(Jan 1957)

Design Memorandum No. 4 - Supplement to Design Memorandum No. 2 - General -
North Springfield Dam & Reservoir (June 1957)
North Springfield Dam and Reservoir Contract Drawings (Feb 1958)

1.4 Project Description

North Springfield Dam is located in east-central Vermont on the Black River at North Springfield, Windsor County. It is about 8.5 miles upstream of the confluence of the Black and Connecticut Rivers and 3 miles northwest of Springfield, Vermont. The project, placed in operation in September 1960, prevents flooding in North Springfield and Springfield and reduces flows in the Connecticut River.

The dam consists of compacted random earth and rock fill approximately 2,940 feet long with a maximum height of 120 feet above the river bed. The elevation of the top of the dam is 570 feet msl. A conventional side channel spillway weir built in rock is located at the left abutment of the dam. The 384-foot long crest, having an ogee shape, is at elevation 545.5 feet.

The outlet works consist of an approach channel, intake structure, discharge conduit and discharge channel. The intake structure, located 321 feet upstream from the axis of the dam, houses the necessary equipment to operate the three 5x12 foot slide gates with hydraulic hoists. A three-span cantilever deck girder bridge connects the intake structure to the roadway on the embankment.

The total drainage area controlled by the project is 158 square miles. The reservoir when filled to spillway crest elevation 545.5 has a total capacity of 51,100 acre-feet, a surface area of 1,200 acres and is about 5.4 miles long. A permanent pool is maintained at elevation 467 feet, except during the summer when the pool is raised to elevation 475 for recreational purposes. Since the completion of the project, the major impoundments have been:

Year	Month	Max. Water Surface Elev. (ft.-msl)	Net Storage (acre-feet)	% of Flood Control Pool Utilized
1969	April	530.8	34,300	69
1973	July	529.5	31,500	65
1976	April	506.2	14,600	29
1967	April	504.3	13,560	27
1972	May	504.0	13,375	27

1.5 Pertinent Hydraulic Data

The hydraulic data used for this review of structural stability are identical to those used in the original design computations made in 1957. The data are as follows:

FULL POOL CONDITION - Reservoir at spillway crest Elevation 545.5; no downstream tailwater in side channel spillway.

DESIGN DISCHARGE CONDITION - Reservoir at SDF maximum surcharge Elevation 564.8; downstream tailwater in side channel spillway at Elevation 532.0

1.6 Discussion of Analysis and Criteria

The principal structural elements which were analyzed for stability consist of the following:

- (a) Spillway Weir
- (b) Right Approach Channel Wall
- (c) Left Approach Channel Wall
- (d) Intake Structure
- (e) Service Bridge Piers and Abutment

The adequacy of sliding resistance of structures subjected to lateral loadings is determined by the use of the shear-friction factor of safety formula as outlined in ETL 1110-2-184 (25 Feb 1974). Sliding stability is evaluated by comparing the available sliding resistance with the lateral force tending to induce sliding. For the spillway weir, right approach channel wall and intake structure, a minimum shear-friction factor of safety of 4.0 is required for all conditions of loading when earthquake is not considered. For loading conditions where earthquake is considered, this factor of safety should exceed 2.67. The left approach channel wall and service bridge piers and abutment should have a factor of safety greater than 1.5 for all loading conditions. Sliding stability was checked by an alternative method in the original design computations made in 1957.

The resistance to overturning is determined according to current criteria by the location of the resultant of vertical forces at the base. The resultant should be located within the middle third of the base for all conditions of loading when earthquake is not considered. For loading conditions where earthquake is considered, it is acceptable if the resultant stays within the base, provided that allowable foundation pressures are not exceeded. For retaining walls founded on rock, the resultant may be outside the middle third, but within the base, if foundation pressures are within allowable values and the factor of safety against sliding is adequate. There have been no significant changes in overturning criteria since the original computations were made.

North Springfield Dam is located in Seismic Zone 2 (moderate damage) as shown on the Seismic Zone Map of Contiguous States, included with ER 1110-2-1806 (30 Apr 77). Therefore, this analysis takes into account earthquake forces induced by accelerations equal to 0.05g. In the original computations, seismic accelerations of 0.10g were used for stability analysis of all structural elements except the intake structure and service bridge piers which were analyzed for an acceleration of 0.05g.

In accordance with EM 1110-2-2200 (23 Nov 1960), the seismic forces applied to this stability analysis are as follows:

- (a) Inertia force P_{el} due to acceleration of the structure, acting through the center of gravity in any direction. $P_{el} = 0.05W$, where, W is the weight of the structure.

(b) Inertia force P_{e2} induced by the impoundment of water. This force was computed using Westergaard's formula, as outlined in EM 1110-2-2200 (23 Nov 1960). The following parameters were used throughout: acceleration equal to $0.05g$, period of vibration equal to 1 second.

(c) Dynamic earth pressure, in accordance with EM 1110-2-2502 (25 Jan 1965), was applied at a distance of $2/3$ the fill height from the base. The pressure is assumed equal to 10 percent of static lateral earth pressure. The backfill between a sloping wall and a vertical plane through the heel was added to the wall mass for computation of inertia force P_{e1} .

No vertical acceleration is considered in this analysis. Uplift is assumed unaffected by earthquake.

The uplift pressure at any point under a structure is the tailwater pressure plus the pressure measured as an ordinate from tailwater to the hydraulic gradient between the upstream and downstream pool. Uplift pressure is considered to act over 100 percent of the base area. Uplift pressure considered in the original computations was equal to the value used in this analysis.

Ice pressure of 10,000 pounds per lineal foot of structure is applied in this analysis in accordance with EM 1110-2-2200 (23 Nov 1960). Ice pressure was also considered in the original computations.

Wind pressure of 30 pounds per square foot is used in this stability investigation and in the original computations.

1.7 Discussion of Foundations and Foundation Parameters

All of the structural elements considered in this stability analysis, except the service bridge abutment, are founded on rock. The service bridge abutment is founded in the random fill of the dam. As described in D.M. No. 2 - General - North Springfield Dam & Reservoir (Jan 1957), the bedrock formation underlying the dam site is a highly metamorphosed schist. The spillway weir, which is secured to the foundation with #11 reinforcing bars that extend 25 feet into rock, is the only structure which is mechanically anchored to the rock foundation.

In the original design computations, an allowable bearing pressure on the schist bedrock of 20 tons per square foot and an allowable bearing pressure on random fill of 2 tons per square foot were used. These allowable values are used in this analysis.

Shearing strength of the schist bedrock was given as 20 kips per square foot in the original computations. The shear strength of the bedrock is greater than the shear strength for the bonded surface between rock and concrete. Therefore, throughout this analysis the critical sliding plane of resistance is assumed to be the plane of contact between the concrete and the rock foundation.

All of the structural elements considered, except the spillway weir, are subjected to lateral forces induced by earth backfill. Earth pressures acting on the intake structure and service bridge piers and abutment are considered to be active pressures. Earth pressures acting on the right and left approach channel walls are considered to be at-rest pressures in accordance with EM 1110-2-2502 (25 Jan 1965).

Foundation parameters used for this analysis are as follows:

- (a) Allowable bearing pressure on rock - 20 tons per square foot (same as value used in original computations).
- (b) Allowable bearing pressure on random fill - 2 tons per square foot (same as value used in original computations).
- (c) Shear at interface between rock and concrete - 80 pounds per square inch (based on ACI 318-71, composite concrete, allowable bond shear stress for clean and intentionally roughened contact surfaces without mechanical anchorages).
- (d) Shear strength of rock - 20 kips per square foot (same as value used in original computations).
- (e) Coefficient of frictional resistance - 0.65 (rock on rock), 0.5 (concrete on rock), 0.45 (concrete on random fill).
- (f) Coefficient of active earth pressure = 0.35 (assumed value).
- (g) Coefficient of at-rest earth pressure = 0.5 (in accordance with EM 1110-2-2502 (25 Jan 1965)).

1.8 Method of Computation

Stability analysis of the spillway weir and right approach channel wall was performed using the computer program "DAMPAC", developed by the Corps of Engineers, New England Division. "DAMPAC", Corps of Engineers Program No. 713 F5 D0 100 and 105, is a fully documented design package for stability analysis of concrete gravity dams.

Stability of the left approach channel wall, intake structure, and service bridge piers and abutment was investigated by manual calculations.

PART II

RESULTS OF THE ANALYSIS

2.1 Spillway Weir

The side channel spillway weir is a low concrete ogee structure with a constant cross section. The spillway is anchored to the foundation with #11 reinforcing bars that extend diagonally into rock. A grout curtain and drains were installed under the base to reduce uplift forces. Stability was investigated at two elevations: El. 535.0, the interface between concrete and rock, and El. 513.35, bottom of the rock anchors.

At El. 535.0, the section analyzed has a height of 10.5 feet and a base length of 26.08 feet. For the analysis of this section, a reduction in uplift, in accordance with EM 1110-2-2220 (23 Nov 1960), is taken to account for the effect of the grout curtain and drains. The drains are assumed to be 50 percent effective in reducing uplift.

At El. 513.35, the section analyzed has a height of 32.15 feet and a base length of 48.17 feet. No reduction in uplift is taken at this elevation.

As outlined in EM 1110-2-2220 (23 Nov 1960), the loading conditions which were considered for the analysis of the spillway weir are as follows:

Case I. Construction Condition. Spillway completed but no water in reservoir, no tailwater, wind load on downstream face.

Case II. Normal Operating Condition. Pool elevation at spillway crest. Minimum tailwater. Ice pressure.

Case III. Induced Surcharge Condition. Pool elevation at top of partially opened spillway gate. (This case is not applicable to this analysis because the spillway at North Springfield Dam is ungated).

Case IV. Flood Discharge Condition. Reservoir at maximum flood pool elevation. Tailwater at flood elevation. Tailwater pressure at 60 percent of full value, except that full value is used for computation of the uplift. No ice pressure.

Case V. Construction Condition with Earthquake. Earthquake acceleration in a downstream direction (thus directing inertia forces upstream). No water in reservoir. No wind load. No tailwater.

Case VI. Normal Operating Condition with Earthquake. Earthquake acceleration in an upstream direction (thus directing inertia forces downstream). Reservoir at spillway crest. Minimum tailwater. No ice pressure.

For Load Cases I through IV, stability criteria are satisfied if the resultant falls within the middle third of the base and the factor of safety against sliding is greater than 4.0. For Load Cases V and VI, stability criteria are satisfied if the resultant stays within the base, provided that allowable foundation pressures are not exceeded, and the factor of safety against sliding is greater than 2.67.

The results of the stability analysis of the spillway weir are contained in Table 1. The criteria for overturning, sliding and foundation pressures are satisfied.

TABLE 1
STABILITY ANALYSIS OF SPILLWAY WEIR

Section (1)	Loading Case	Location of Resultant		Sliding Factor of Safety (2)	Length of Base in Bearing (ft)	Bearing Pressure on Rock Kips/S.F.	
		In Middle Third	In Base			Upstream	Downstream
Sect. above El. 535.0 Ht = 10.5 ft. Base = 26.08 ft. El. 513.35 Ht = 32.15 ft. Base = 48.17 ft. ∞	I	Yes	-	997.2	26.08	1.44	0.65
	II	Yes	-	23.2	26.08	0.16	1.66
	III	Case III not applicable					
	IV	Yes	-	19.3	26.08	0.49	1.08
	V	Yes	-	230.3	26.08	1.48	0.62
	VI	Yes	-	62.5	26.08	1.03	0.79
Sect. above El. 513.35 Ht = 32.15 ft. Base = 48.17 ft. El. 513.35 Ht = 32.15 ft. Base = 48.17 ft. ∞	I	Yes	-	1123.8	48.17	5.83	1.81
	II	Yes	-	25.0	48.17	2.45	3.41
	III	Case III not applicable					
	IV	Yes	-	14.8	48.17	1.34	3.38
	V	Yes	-	117.8	48.17	6.06	1.59
	VI	Yes	-	24.4	48.17	2.96	2.90

(1) See Plates 2 and 3 for details.

(2) Factor of safety computed with the following parameters: for stability at El. 535.0, bond shear value of 80 psi and coefficient of friction of 0.5; for stability at El. 513.35, shear value of 139 psi (=20 ksf) and coefficient of friction of 0.65.

2.2 Right Approach Channel Wall

The right approach channel wall is a gravity type wall founded on bedrock. It is approximately 245 feet long and it completes the closure between the spillway weir and the earthfill dam. The top of the wall contains a roadway which is a continuation of the roadway on the dam. The crown of the roadway, which is an integral part of the wall, is at Elevation 570 msl.

Three representative sections of the wall were analyzed: Section 1-1 (34.19 ft. height, 22.61 ft. base) near the spillway weir; Section 6-6 (33.19 ft. height, 22.55 ft. base) approximately 26 feet upstream from the spillway crest; and Section 11-11 (45.69 ft. height, 23.77 ft. base) enveloped by the embankment. Since the wall is in effect an extension of the dam, the loading conditions and criteria by which it was evaluated are those required by EM 1110-2-2200 (23 Nov 1960) for concrete gravity dams. The load cases and criteria are identical to those used for the spillway weir.

Table 2 contains the results of the stability analysis of the right approach channel wall. All stability criteria are satisfied for the three sections considered.

TABLE 2

STABILITY ANALYSIS OF RIGHT APPROACH CHANNEL WALL

<u>Section (1)</u>	<u>Loading Case</u>	<u>Location of Resultant</u>		<u>Sliding Factor of Safety (2)</u>	<u>Length of Base in Bearing (ft)</u>	<u>Bearing Pressure on Rock Kips/S.F.</u>	
		<u>In Middle</u>	<u>Third</u>			<u>Upstream</u>	<u>Downstream</u>
Sect. 1-1 Ht = 34.19 ft. Base = 22.61 ft.	I	Yes		-	286.2	22.61	5.78 0.07
	II	Yes		-	20.5	22.61	3.36 1.79
	III			Case III not applicable			
	IV	Yes		-	25.0	22.61	3.62 1.03
	V	No		Yes	88.8	21.59	6.13 0
	VI	Yes		-	37.0	22.61	4.15 1.01
Sect. 6-6 Ht = 33.19 ft. Base = 22.55 ft.	I	Yes		-	27.0	22.55	5.84 0.93
	II	Yes		-	87.6	22.55	3.64 2.49
	III			Case III not applicable			
	IV	Yes		-	24.6	22.55	1.92 3.30
	V	Yes		-	21.5	22.55	6.25 0.52
	VI	Yes		-	92.9	22.55	4.35 1.77
Sect. 11-11 Ht = 45.69 ft. Base = 23.77 ft.	I	Yes		-	86.1	23.77	8.01 4.71
	II	Yes		-	19.6	23.77	3.00 8.29
	III			Case III not applicable			
	IV	Yes		-	10.7	23.77	0.03 9.85
	V	Yes		-	34.7	23.77	9.43 3.28
	VI	Yes		-	25.2	23.77	4.38 6.91

(1) See Plates 6, 7, and 8 for details.

(2) Factor of safety calculated for bond shear value of 80 psi, coefficient of friction of 0.5.

2.3 Left Approach Channel Wall

The left approach channel wall is an "L"-shaped semi-gravity type wall keyed 3 feet into bedrock. It is approximately 150 feet long and retains earth backfill at the left abutment of the spillway weir.

The top of the wall perpendicular to flow is at El. 570 and parallel to flow slopes to the top of the rock at El. 550 in approximately 40 feet.

As outlined in EM 1110-2-2400 (2 Nov 1964) in the section entitled "Approach Channel Walls," the loading conditions which were considered for analysis of the left approach channel wall are as follows:

Case I. Channel empty. Backfill naturally drained.

Case II. Partial sudden drawdown of reservoir from design flood level. Water in channel to drawdown elevation which may occur suddenly. Fill submerged to profile reached during design flood, drained above. (For this analysis, consider drawdown from El. 564.8 to El. 545.5).

Case III. Sudden rise of reservoir to design flood elevation. Water in channel to design flood elevation. Fill submerged to concurrent water surface in fill, naturally drained above.

Case IA. Same as Case I with earthquake load added.

For all load cases considered, stability criteria are satisfied if the resultant stays within the base, provided that allowable foundation pressures are not exceeded, and the factor of safety against sliding is greater than 1.5.

Two sections of the wall were analyzed: Section C-C (30 ft. height, 19 ft. base) taken through the portion of wall perpendicular to flow and Section G-G (19.13 ft. height, 11.9 ft. base) taken through the portion of wall parallel to flow. Section G-G was analyzed only for Load Cases I and IA.

Table 3 contains the results of the stability analysis of the left approach channel wall. All stability criteria are satisfied for the two sections considered.

TABLE 3

STABILITY ANALYSIS OF LEFT APPROACH CHANNEL WALL

<u>Section (1)</u>	<u>Loading Case</u>	<u>Location of Resultant</u>		<u>Sliding Factor of Safety (2)</u>	<u>Length of Base in Bearing (ft)</u>	<u>Bearing Pressure on Rock Kips/S.F.</u>	
		<u>In Middle Third</u>	<u>In Base</u>			<u>Toe</u>	<u>Heel</u>
Sect. C-C Ht = 30 ft. Base = 19 ft.	I	Yes	-	12.7	19.00	6.83	0.03
	II	No	Yes	6.0	3.69	27.34	0
	III	No	Yes	10.2	10.47	7.82	0
	IA	No	Yes	10.0	14.79	8.82	0
Sect. G-G Ht = 19.13 ft. Base = 11.9 ft.	I	No	Yes	19.1	11.61	4.79	0
	IA	No	Yes	15.0	9.03	6.16	0

(1) See Plates 4 and 5 for details.

12 (2) Factor of safety calculated for bond shear value of 80 psi, coefficient of friction of 0.5.

2.4 Intake Structure

The intake structure is a monolithic tower consisting of a control house, shaft and gated water passages. The height of the reinforced concrete structure from the roof top of the control house to the bottom of the base slab (El. 448.0) is 157.37 feet. Due to the position of the intake structure, i.e., its embedment in rock on the sides up to Elevation 490.0, it was determined that stability in only the upstream-downstream direction need be investigated.

As outlined in EM 1110-2-2400 (2 Nov 1964), the loading conditions which were considered for analysis of the intake structure are as follows:

Case I. Reservoir empty. Wind load to produce most severe foundation pressures.

Case II. Gate structure with all gates open. Reservoir at spillway crest. Ice pressure. Uplift. Water surface inside structure drawn down to hydraulic gradient with all gates open.

Case III. Similar to Case II, except that gate structure operating with one outside gate closed, others open.

Case IV. Gate structure with gates closed. No flow in conduits. Reservoir at spillway crest. Ice pressure. Uplift. Structure full of water upstream from closed gates.

Case V. Reservoir raised to spillway design flood level for whichever of preceding Cases II, III, or IV is most critical. No ice pressure. (For this analysis, Case II is most critical).

Case VI. Bulkheads in place. Reservoir at maximum level at which bulkheads are used. (For this analysis, reservoir at El. 490.0).

Case IA, IIA, IIIA, or IVA. Same as Case I, II, III, or IV, respectively, with earthquake load added, except that earthquake is substituted for wind in Case IA, and for ice in the other cases. (The water in the gate structure was added to the mass of the structure for computation of inertia forces).

For Load Cases II, III, IV and VI, stability criteria are satisfied if the resultant falls within the middle third of the base and the factor of safety against sliding is greater than 4.0. For Load Cases I and V, stability criteria are satisfied if 75 percent of the base is in compression and the factor of safety against sliding is greater than 4.0. For Load Cases IA, IIA, IIIA, and IVA, stability criteria are satisfied if the resultant stays within the base, provided that allowable foundation pressures are not exceeded, and the factor of safety against sliding is greater than 2.67.

The results of the stability analysis of the intake structure are contained in Table 4. Stability criteria are satisfied for all the specified loading conditions.

TABLE 4
STABILITY ANALYSIS OF INTAKE STRUCTURE

<u>Section (1)</u>	<u>Loading Case</u>	<u>Location of Resultant</u>	<u>Sliding Factor of Safety (2)</u>	<u>Length of Base in Bearing (ft)</u>	<u>Bearing Pressure on Rock Kips/S.F.</u>	
		<u>In Middle Third</u>	<u>In Base</u>		<u>Upstream</u>	<u>Downstream</u>
Sect. parallel to flow	I	Yes	-	46.1	59.82	10.58 4.33
	II	Yes	-	6.7	59.82	2.34 5.78
Ht = 157.37 ft.	III	Case III not checked - approximately similar to Case II				
Base = 59.82 ft.	IV	Yes	-	6.7	59.82	2.45 5.30
	V	Yes	-	5.8	59.82	2.76 4.27
	VI	Yes	-	24.5	59.82	7.05 3.11
	IA	Yes	-	24.2	59.82	11.88 2.94
	IIA	Yes	-	5.3	59.82	0.49 7.75
	IIIA	Case IIIA not checked - approximately similar to Case IIA				
	IVA	Yes	-	5.3	59.82	0.59 7.27

¹

(1) See Plates 9, 10, and 11 for details.

(2) Factor of safety calculated for bond shear value of 80 psi, coefficient of friction of 0.5.

2.5 Service Bridge Piers and Abutment

Access to the intake structure from the embankment is provided by the three-span service bridge which is approximately 290 feet in length. The embankment end of the bridge is supported by a reinforced concrete abutment with spread footings. The intermediate spans are supported on "T"-shaped piers. Pier No. 2, nearest the intake structure, is founded on bedrock while Pier No. 1, nearest the embankment, is supported on piling driven to bedrock.

The following sections were analyzed: Section F-F (16.02 ft. height, 9.5 ft. base) taken at the centerline of the abutment and the typical section (61.91 ft. height, 18.0 ft. base) of Pier No. 2. Pier No. 1 was not checked in this analysis since it is supported by a system of battered and vertical piles and is approximately half the height of Pier No. 2; therefore, Pier No. 1 was not considered critical for overturning and earthquake forces with respect to current criteria.

The loading conditions which were considered are as follows:

Case I. Dead load reaction of bridge. No water. Drained backfill.
Wind load (for piers only).

Case II. Dead load reaction of bridge. Water level at spillway crest.
Uplift. Wind load (for piers only).

Case III. Dead load reaction of bridge. Floodwater level to El. 564.8.
Uplift.

Case IA, IIA. Same as Case I or II plus earthquake.

For Load Cases I through III, stability criteria are satisfied if the resultant falls within the middle third of the base and the factor of safety against sliding is greater than 1.5. For Load Cases IA and IIA, stability criteria are satisfied if the resultant stays within the base and the factor of safety against sliding is greater than 1.5. Allowable foundation pressures are not to be exceeded for any load cases.

Table 5 contains the results of the stability analysis of the service bridge piers and abutment. All stability criteria are satisfied.

TABLE 5

STABILITY ANALYSIS OF SERVICE BRIDGE PIERS AND ABUTMENT

Section (1)	Loading Case	Location of Resultant		Sliding Factor of Safety (2)	Length of Base in Bearing (ft.)	Bearing Pressure Kips/S.F. (3)	
		In Middle Third	In Base			Kips/S.F. Upstream	Kips/S.F. Downstream
Sect. F-F at centerline of abutment Ht = 16.02 ft. Base = 9.5 ft.	I II III IA IIA	Yes Case II identical to Case I Yes Yes	- - - - -	3.1 2.7 2.2	9.5 9.5 9.5	3.15 2.50 3.83	1.91 1.42 1.23
Typ. Sect. of Pier No. 2 Ht = 61.91 ft. Base = 18.0 ft.	I II III IA IIA	Yes Yes Yes Yes Yes	- - - - -	121.7 255.1 630.5 36.8 32.2	18.0 18.0 18.0 18.0 18.0	4.44 4.45 3.59 8.57 0.43	6.52 3.33 3.57 2.39 7.35

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(1) See Plates 12, 13, and 14 for details.

(2) Factor of safety computed with the following parameters: for Sect. F-F, bond shear value of 0 and coefficient of friction of 0.45; for Typ. Sect. of Pier No. 2, bond shear value of 80 psi and coefficient of friction of 0.5.

(3) Abutment is founded on the random fill of the dam. Pier No. 2 is founded on rock.

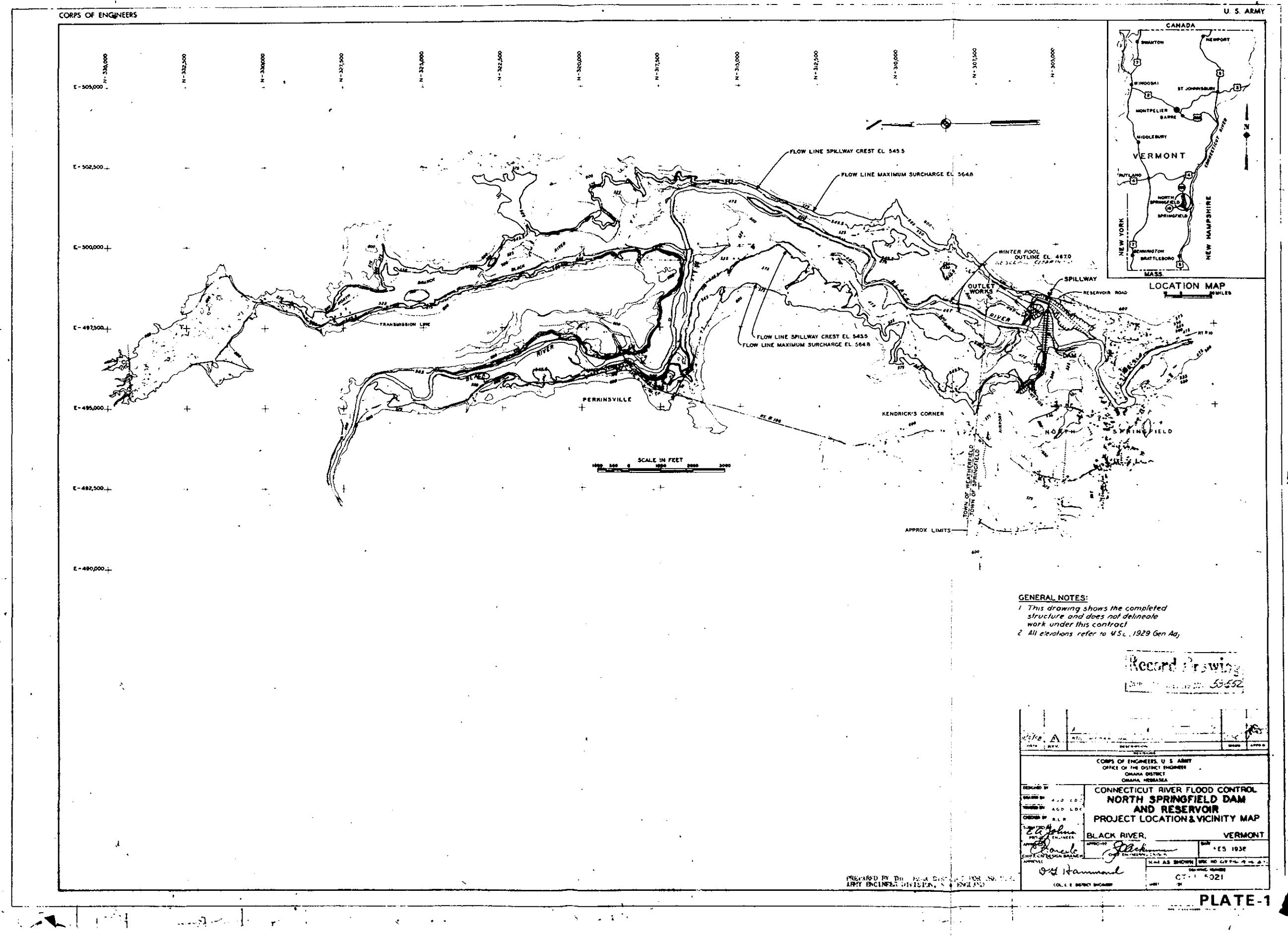
2.6 Conclusions

The following concrete structures at North Springfield Dam were analyzed for stability: spillway weir, right approach channel wall, left approach channel wall, intake structure, and service bridge piers and abutment. All of the structures satisfy the requirements of the current criteria and no modifications or strengthening is required.

APPENDIX A

SELECTED RECORD DRAWINGS

Drawing No.	Title	Plate No.
CT-1-5021, Sh. No. 2	Project Location & Vicinity Map	1
CT-1-5051, Sh. No. 16	Spillway - General Plan and Section	2
CT-1-5052, Sh. No. 17	Spillway - Crest Structure - Concrete & Reinforcement Details	3
CT-1-5055, Sh. No. 20	Spillway - Upstream Abutment - Structural Arrangement	4
CT-1-5056, Sh. No. 21	Spillway - Left Approach Wall - Concrete Details	5
CT-1-5058, Sh. No. 23	Spillway - Downstream Abutment - Structural Arrangement and Details	6
CT-1-5060, Sh. No. 25	Spillway - Right Approach Wall - Plan & Elevation	7
CT-1-5061, Sh. No. 26	Spillway - Right Approach Wall - Sections	8
CT-1-5065, Sh. No. 30	Outlet Works - Plan and Profile	9
CT-1-5066, Sh. No. 31	Outlet Works - Intake Structure - General Arrangment - Sh. 1	10
CT-1-5068, Sh. No. 33	Outlet Works - Intake Structure - Gate Structure - Conc. Details - Sh. 1	11
CT-1-5101, Sh. No. 65	Outlet Works - Service Bridge - General Plan and Elevation	12
CT-1-5102, Sh. No. 66	Outlet Works - Service Bridge - Abutment Details	13
CT-1-5103, Sh. No. 67	Outlet Works - Service Bridge - Piers No. 1 and 2 Details	14



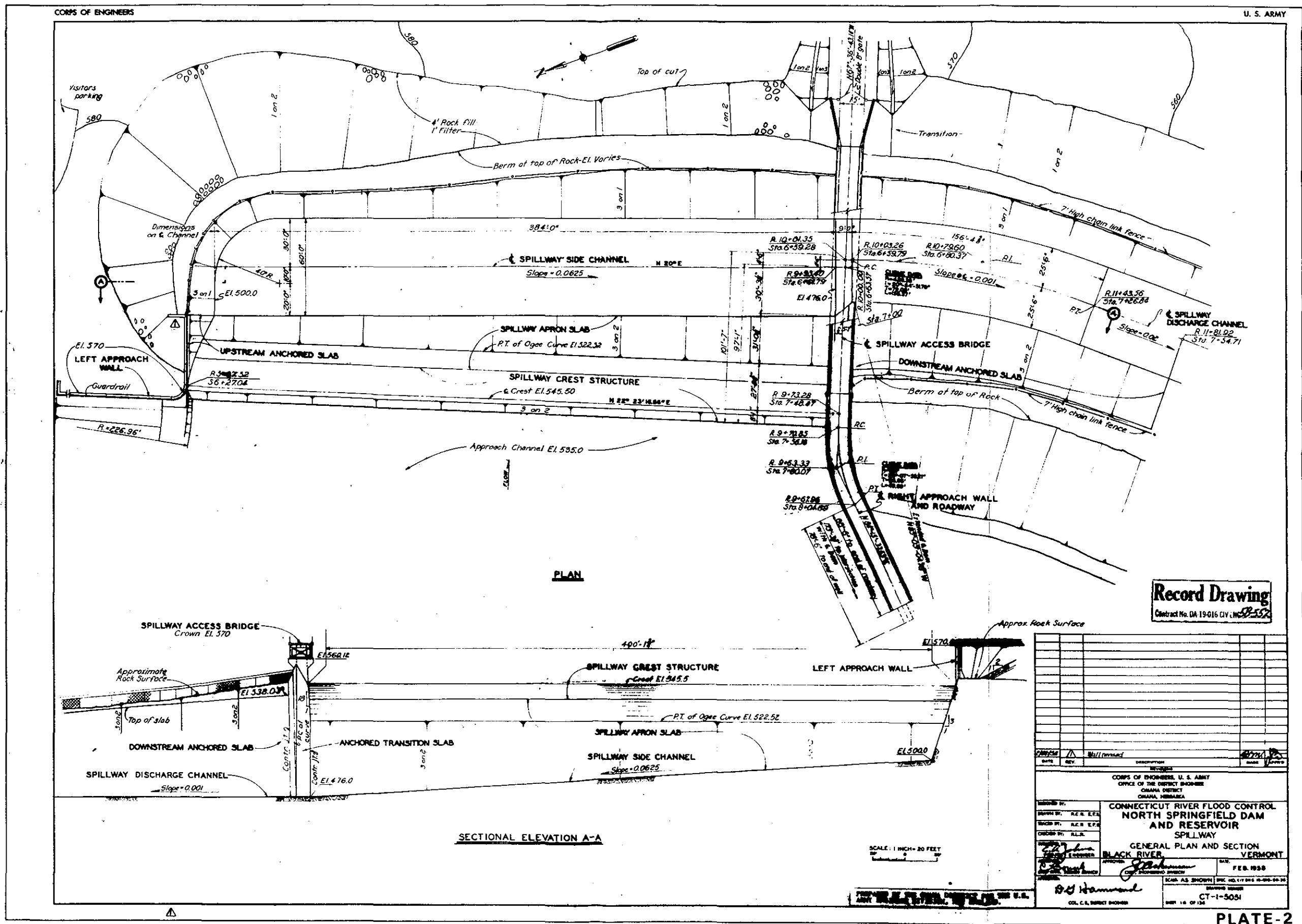


PLATE-2

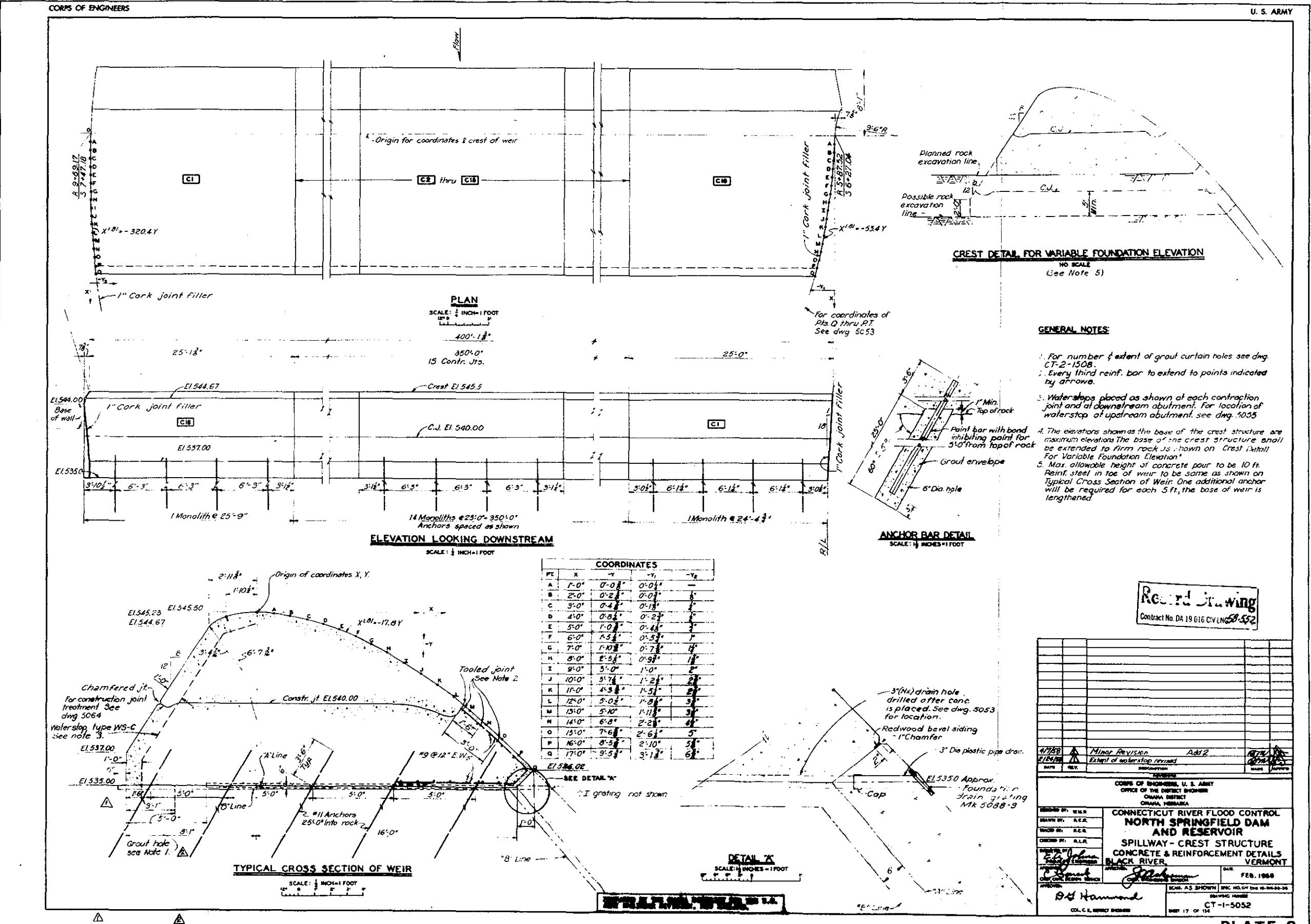


PLATE-3

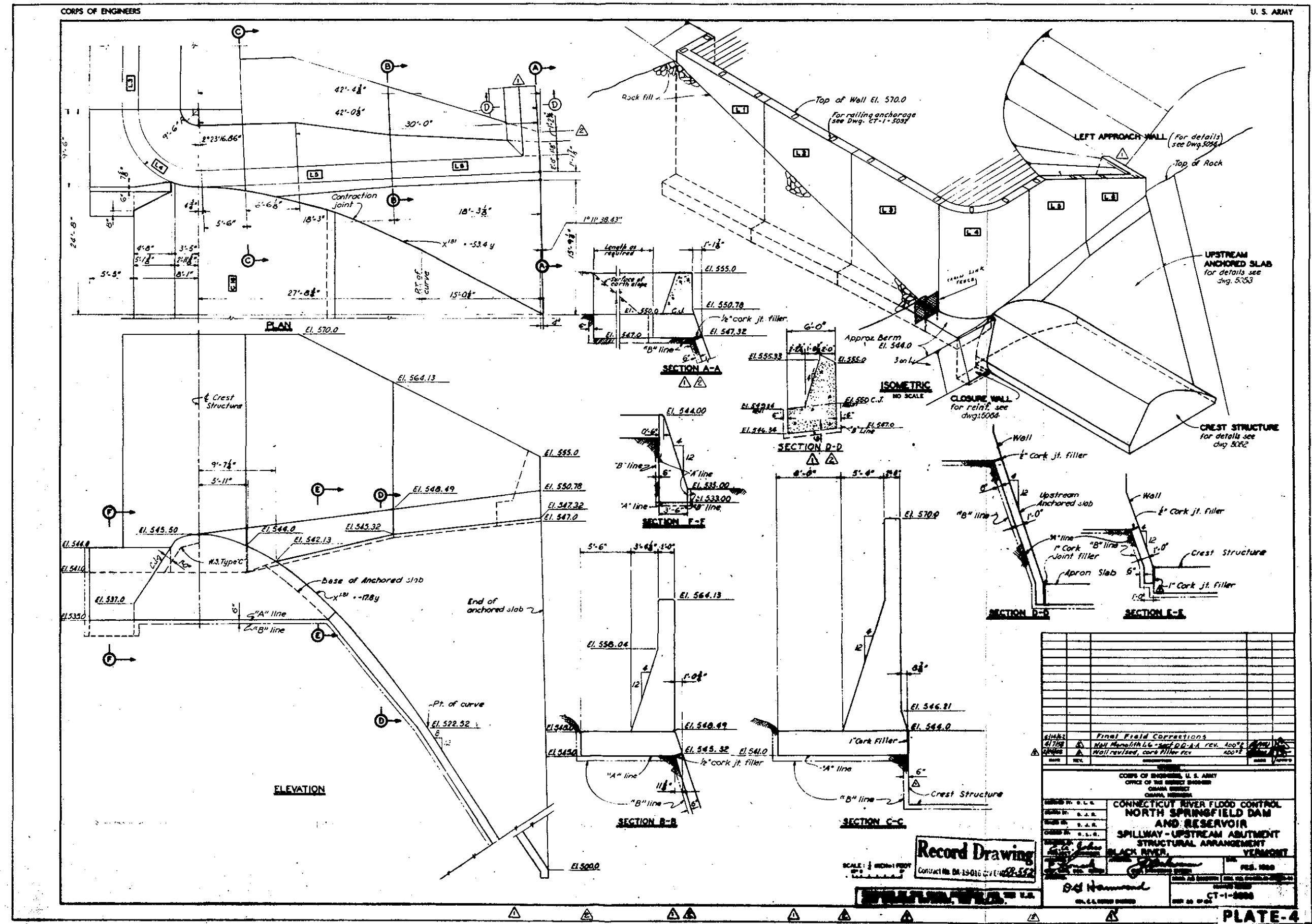
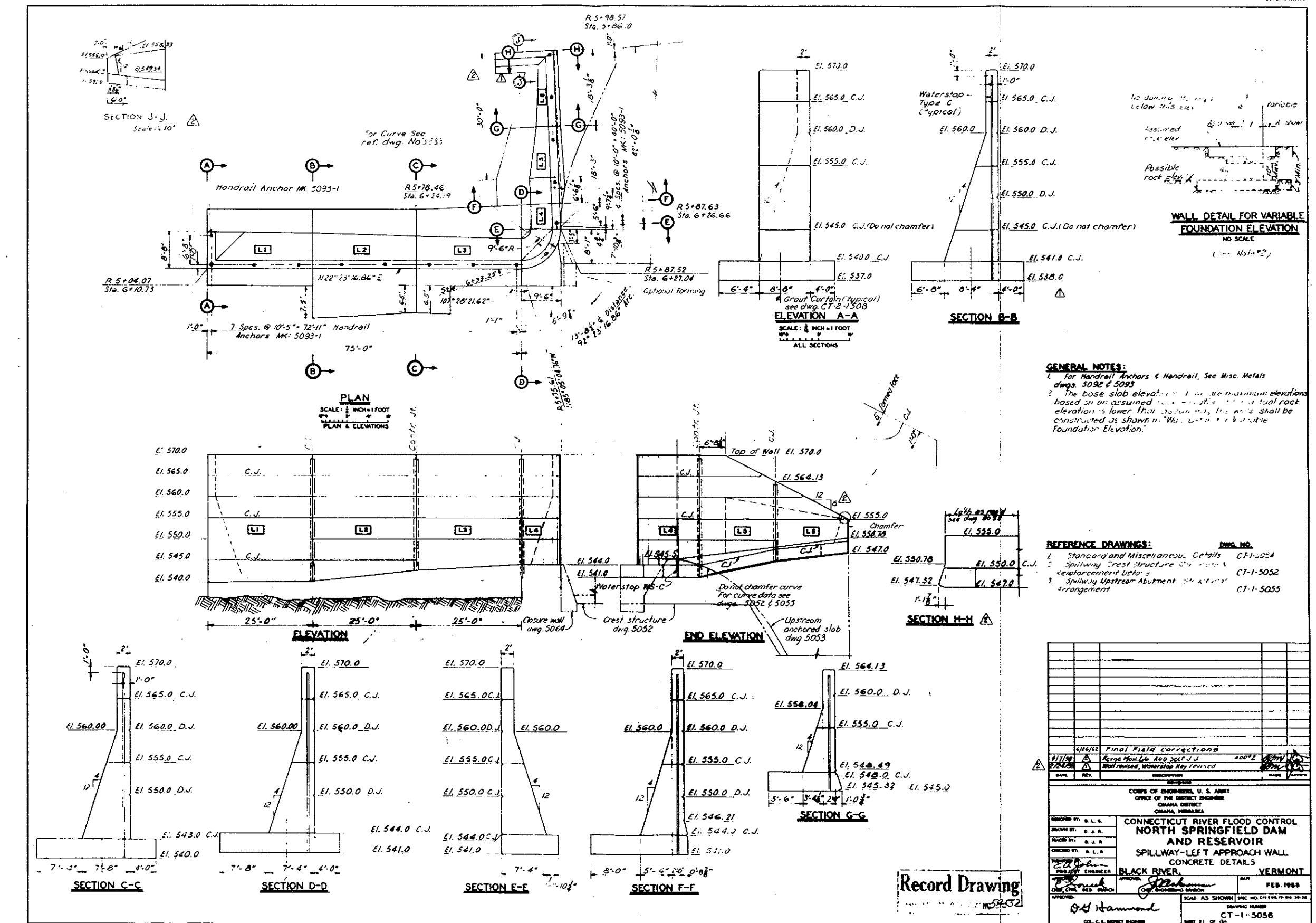
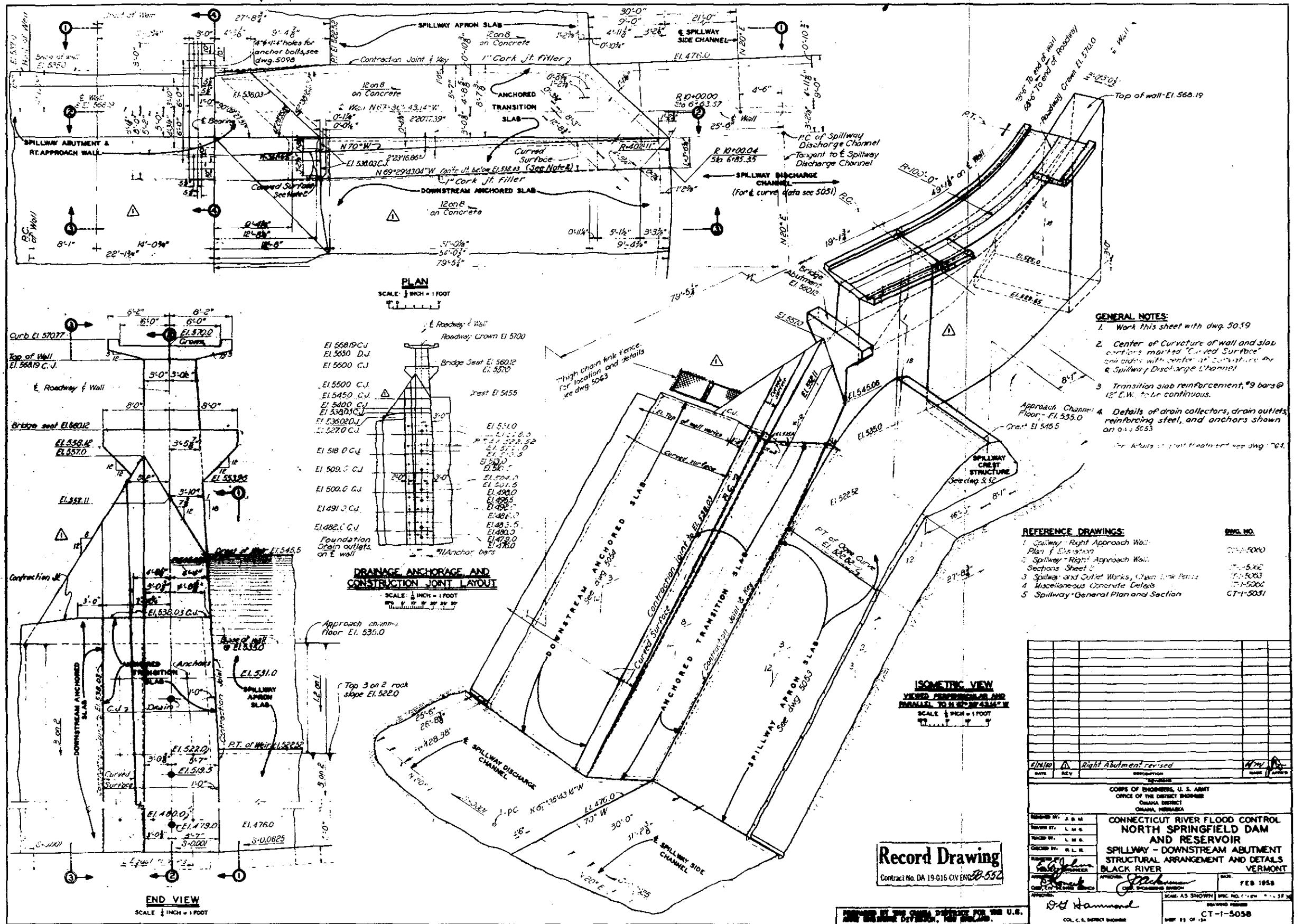
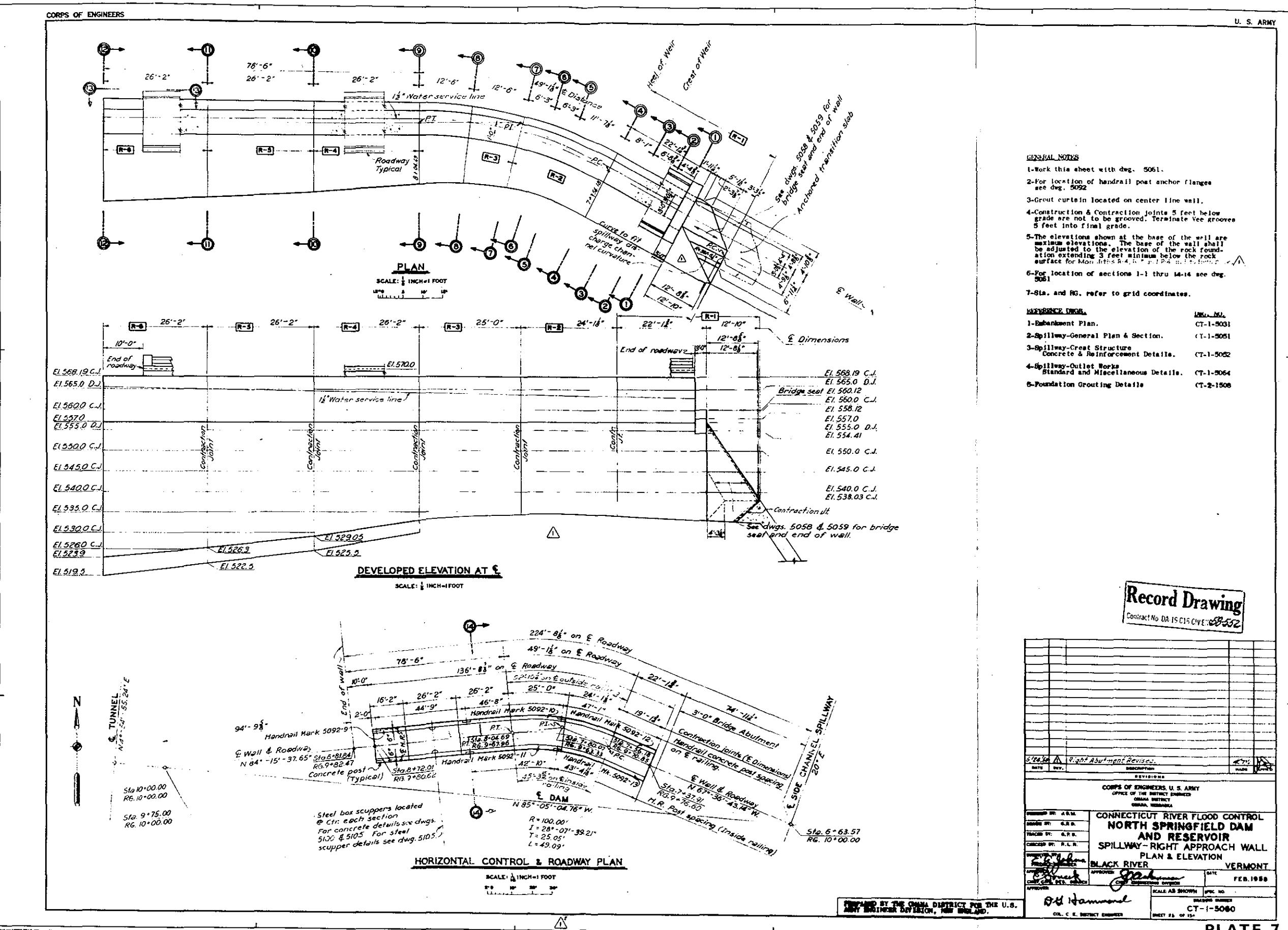
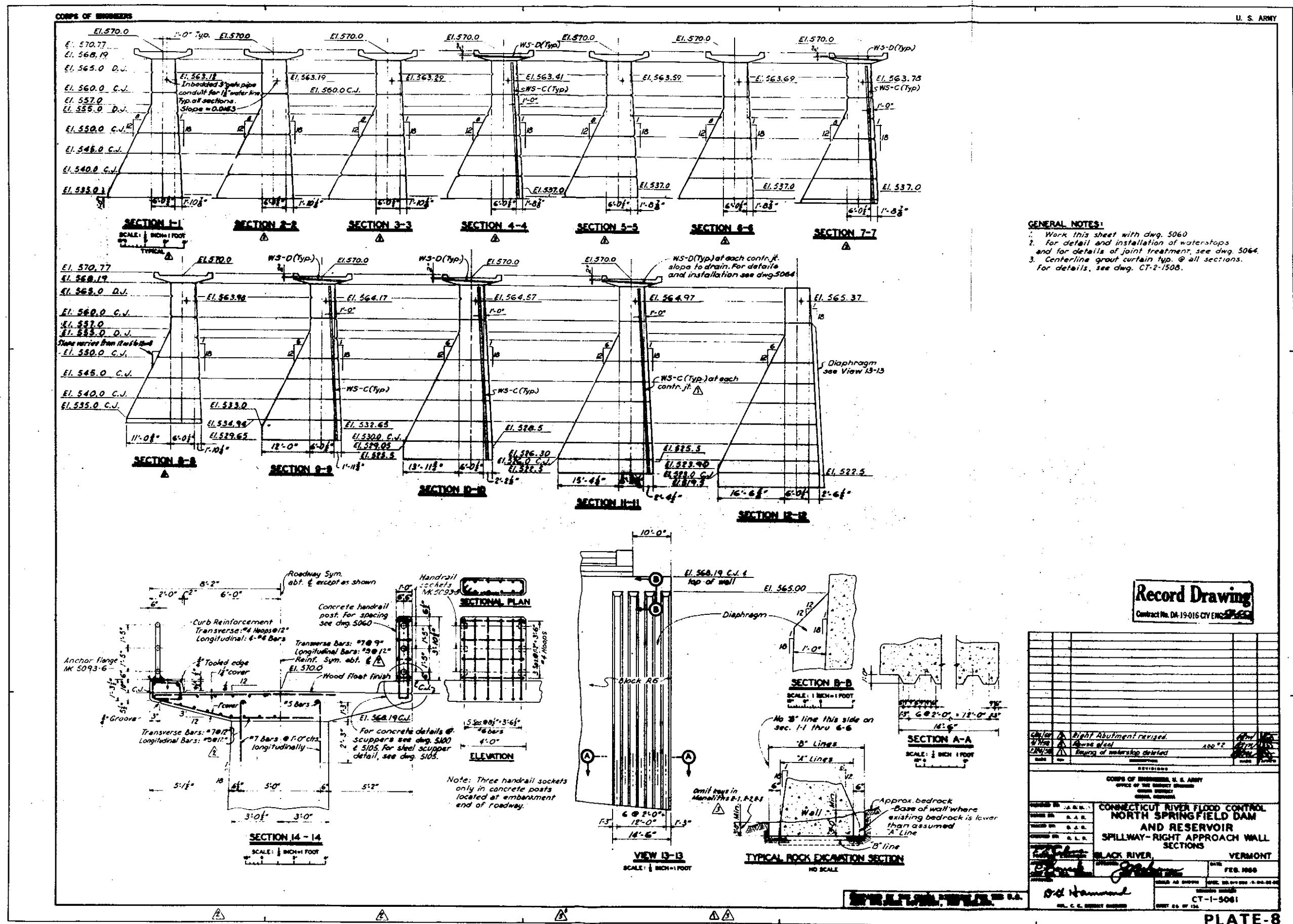


PLATE-4



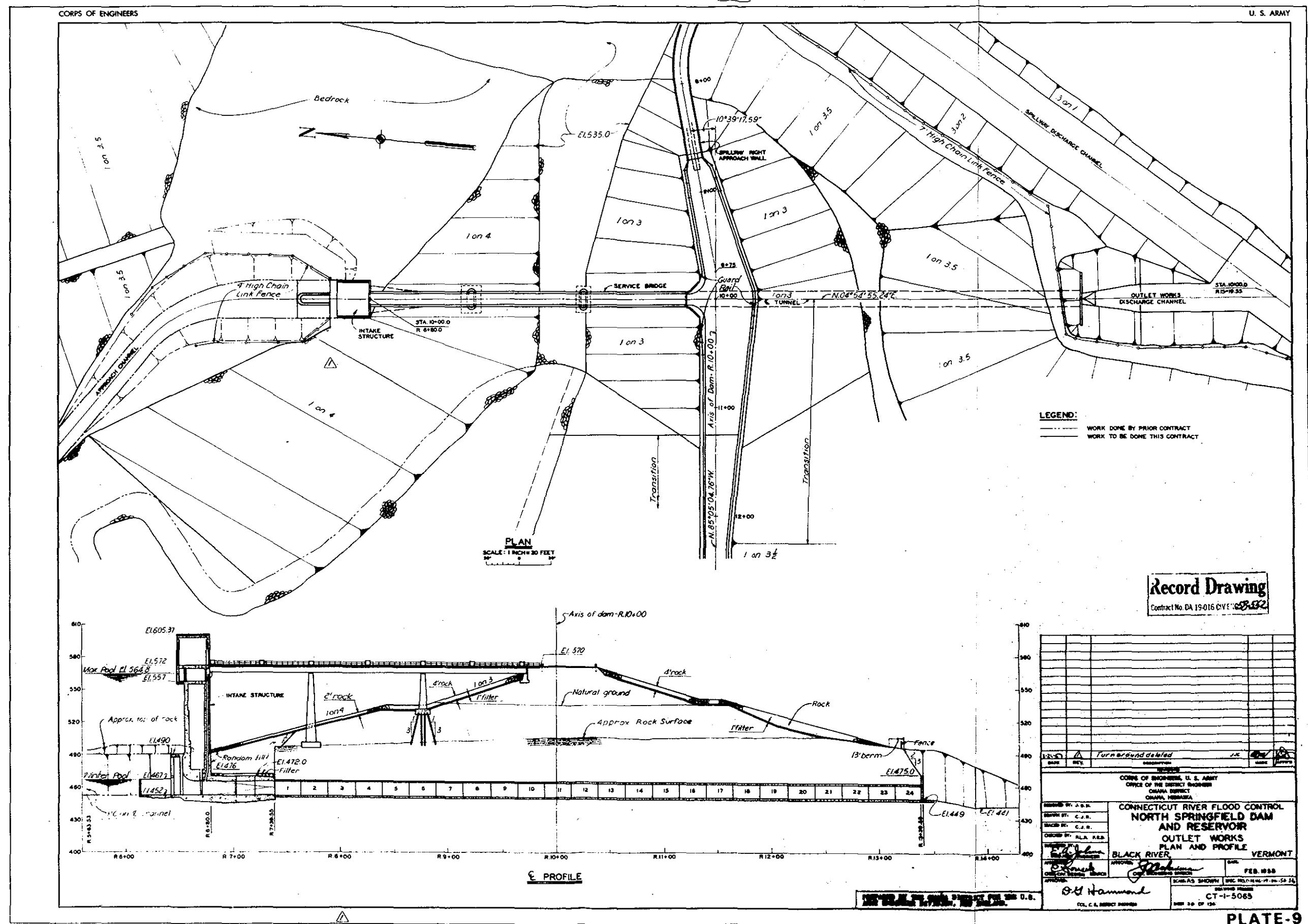


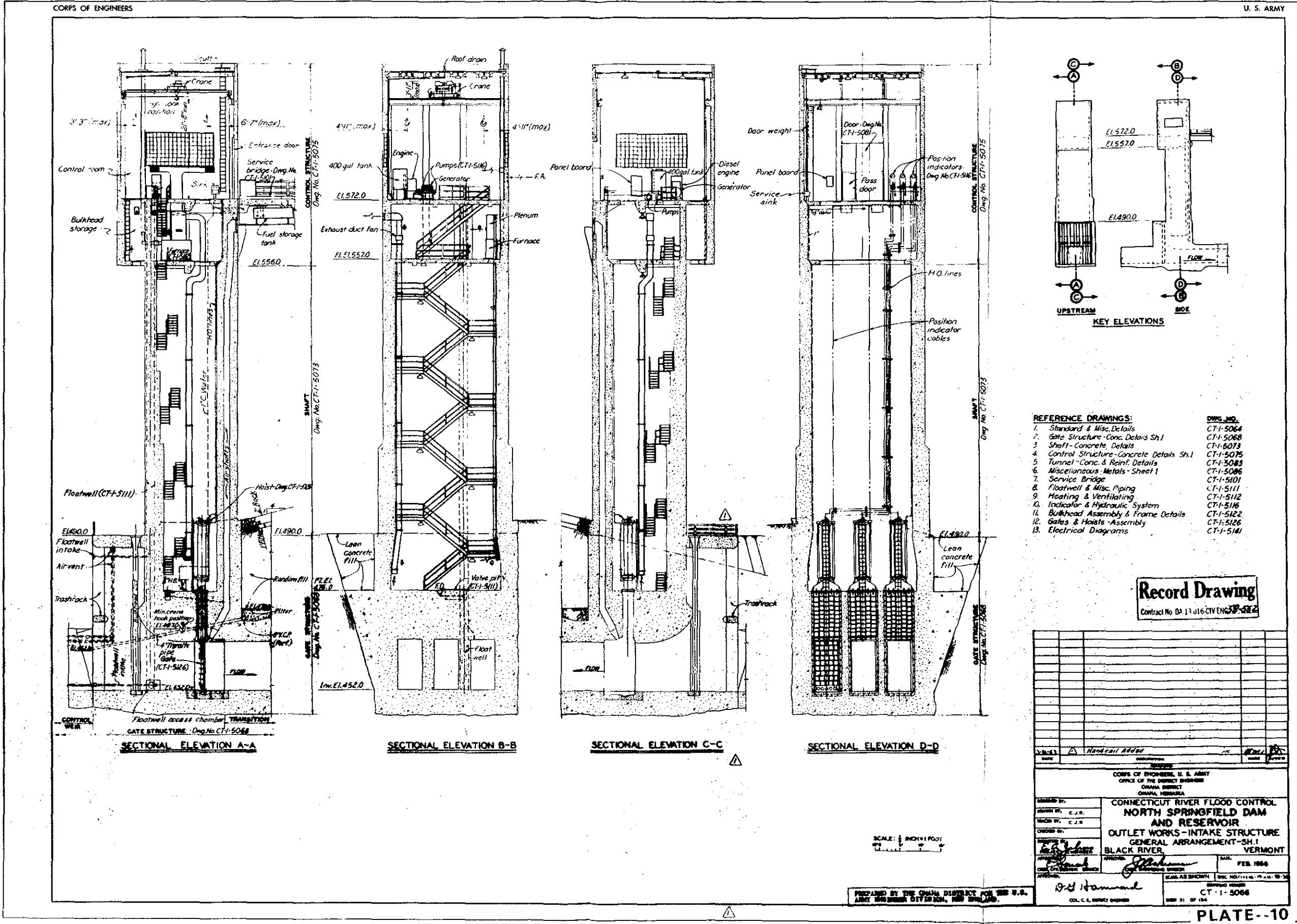




Record Drawing

Contract No. DA-19-016-CIV ENG-~~2~~-400





Record Drawing

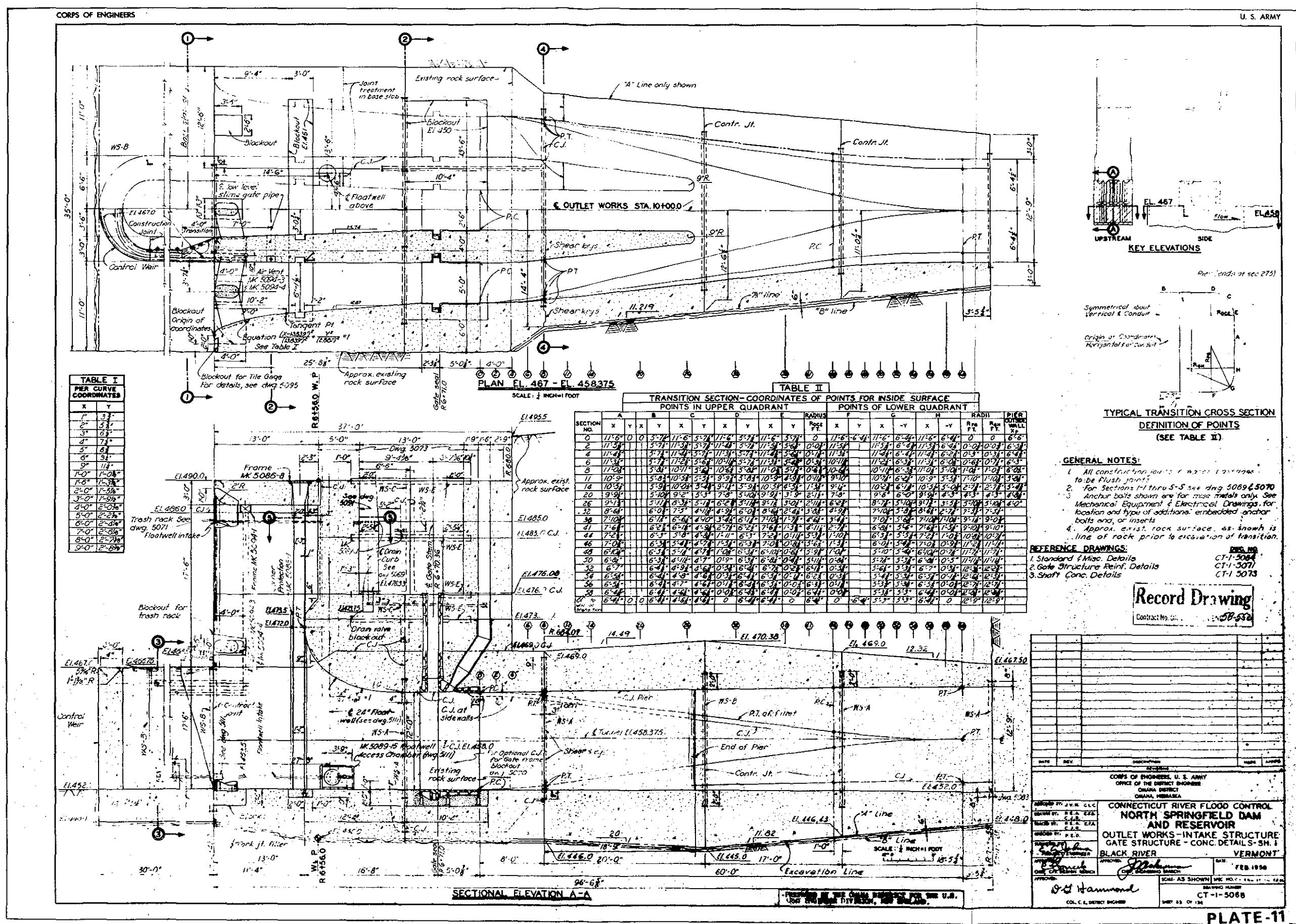
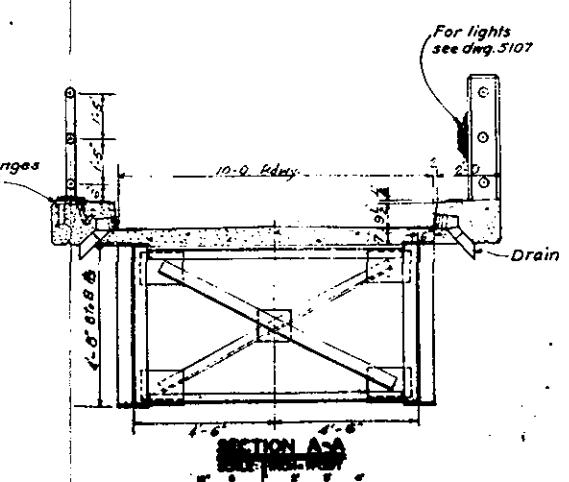
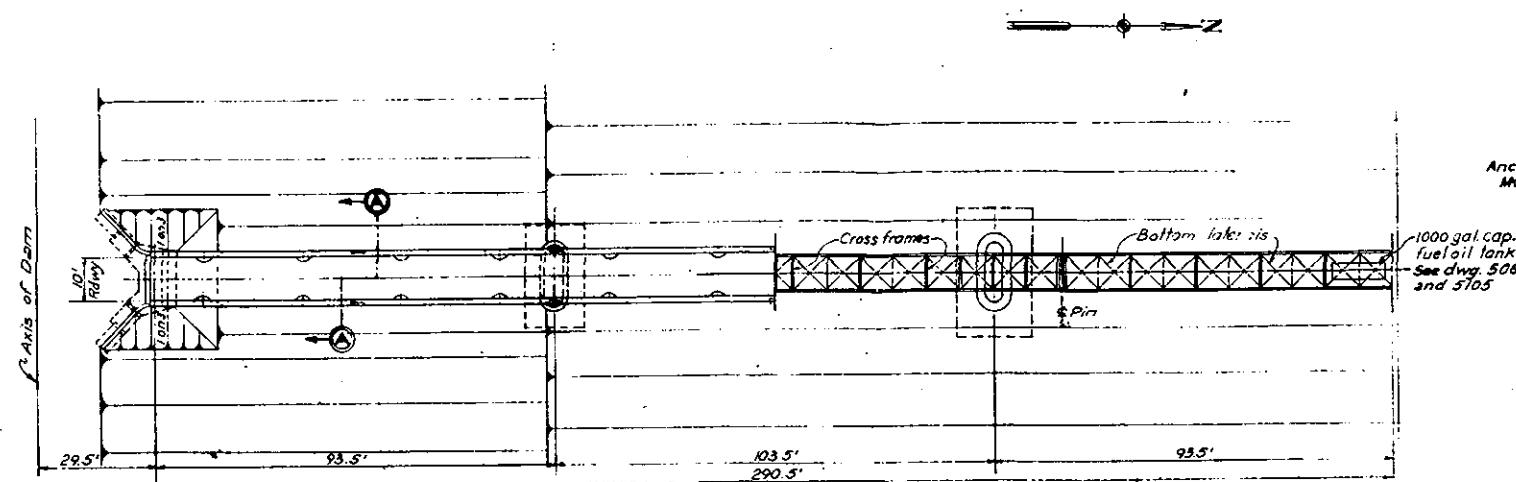
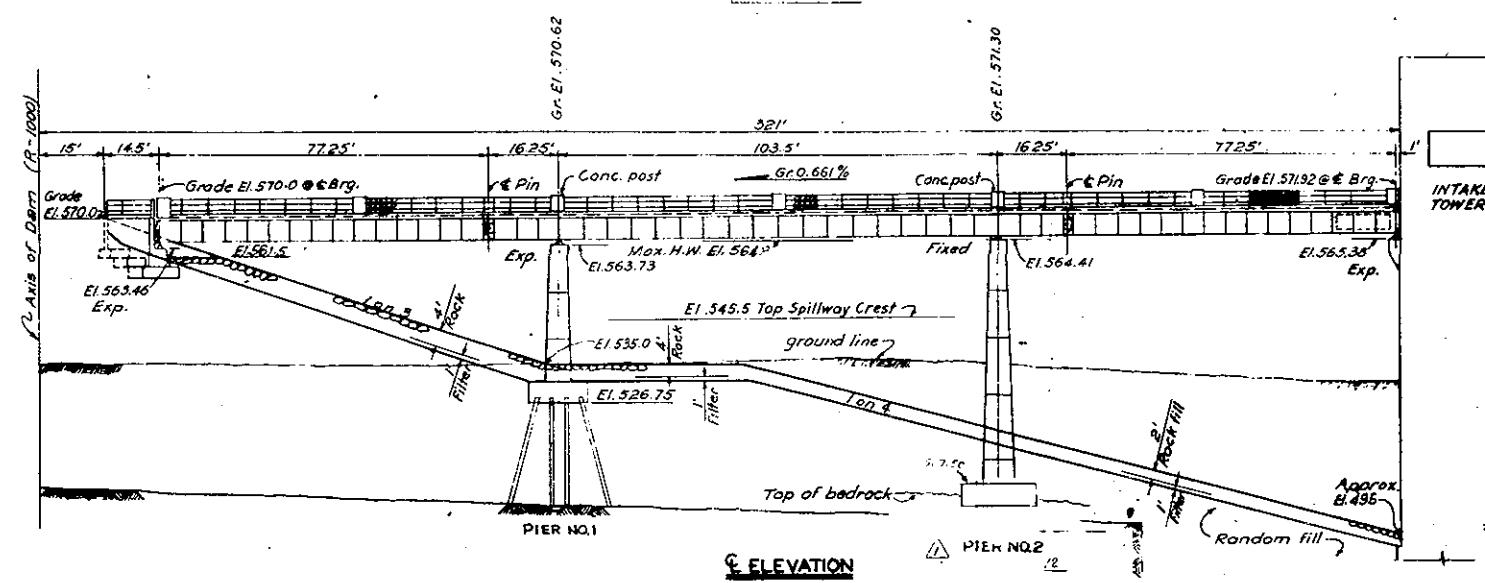


PLATE-11



PARTIAL PLAN

SCALE: 1 INCH = 15 FEET



ELEVATION

ELEVATION

GENERAL NOTES

- This bridge is designed for the following loads at normal elevation in accordance with AASHTO Design Specification - 1959 Edition:
 - Dead load as specified - 1/4" slab thickness allowed for wearing surface on roadway.
 - Live load - standard AASHTO-H15 Loading.
 - All exposed edges of concrete shall be chamfered 1" except as shown.
 - Structural steel.
 - All steel to be welded. In girders, expansion plates, sheathing and as required elsewhere shall be A.S.T.M. A373-54T with the same working stresses as for A.S.T.M. A7 steel.
 - All other steel is to be A.S.T.M. A7.
 - Design loads on piles.
 - The basic design load for axial loading for 10" dia. piles is 37 tons. Loads on piles may be increased for combination loading as per A.A.S.H.O. specs. All piles to be driven to refusal in rock.
 - Threads on pins to be barred on ends to prevent turning.
 - All elevations shown refer to M.S.L. 1929 Gen. Adj.
 - For handrail details see dwg. 5091 and 5092.
 - For electrical details see dwg. 5107.
 - All welded work shall be in accordance with standard Specifications for Welded Highway and Railway Bridges, American Welding Society, 1958.
 - Concrete shall have the following properties:
 - Superstructure $f'_c = 1600$ p.s.i., $n = 8$, $f_b = 1600$ p.s.i.
 - Substructure $f'_c = 3000$ p.s.i., $n = 10$, $f_b = 1600$ p.s.i.
 - Reinforcement shall be staggered here meeting A.S.T.M. A36, intermediate span over hearth slab, A.S.T.M. A16-60, $f_y = 20,000$ p.s.i.
 - Debonding value of compacted reinforcement will be 4000 p.s.i.

